Revised Report
Geotechnical Engineering Services
Proposed Property Development
Multi- and Single-Family Residential Housing
905 Newport Way NW
King County Tax Parcel No. 2824069011
Issaquah, Washington

December 31, 2013
ICE File No. 0520-002

Prepared For:
Vineyards Construction, LLC

Prepared By:
Icicle Creek Engineers, Inc.

December 31, 2013

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Icicle Creek Engineers is pleased to submit two copies of our *Revised Report, Geotechnical Engineering Services, Proposed Property Development, Multi- and Single-Family Residential Housing, 905 Newport Way NW, King County Tax Parcel No. 2824069011, Issaquah, Washington*. Icicle Creek Engineers' services were completed in general accordance with our Proposal dated March 5, 2012 and were authorized in writing by Robert Wenzl on March 6, 2012.

Comments (dated November 8, 2013) by the City of Issaquah on the original report (dated May 14, 2012) were transmitted to ICE by Bob Wenzl on November 12, 2013. ICE responded to these comments on November 21, 2013. Subsequently, Brian Beaman of ICE and Doug Schlepp of the City of the Issaquah discussed the comments and it was agreed that the original report would be revised to include ICE's response to comments. The responses to review comments have been incorporated in this revised report which was prepared and submitted at the request of Mr. Wenzl.

It has been our pleasure to be of service to Vineyards Construction, LLC on this project. If you have any questions regarding the contents of this report or if we can be of further service, please contact us.

Yours very truly,

Icicle Creek Engineers, Inc.

B. R. Beaman

Brian R. Beaman, PE, LEG, LHG
Principal Engineer/Geologist/Hydrogeologist

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Attachment

Submitted via email (PDF) and surface mail

cc: Robert Stevens, Core Design (PDF)

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ISSAQUAH, WASHINGTON

1.0 INTRODUCTION

This revised report presents the results of Icicle Creek Engineers (ICE's) preliminary geotechnical engineering services regarding a proposed residential development at King County Tax Parcel No. 2824069011 in Issaquah, Washington. The property is shown relative to nearby features on the Vicinity Map, Figure 1. The layout of the property is shown on the Site Plan, Figure 2.

2.0 PROJECT DESCRIPTION

2.1 GENERAL

Our understanding of the project is based on discussions with Robert Wenzl of Vineyards Construction, LLC, Robert Stevens, PE of Core Design and Leonard Milbrandt, A/A of Milbrandt Architects (MA) at a meeting on February 28, 2012. For the purpose of this revised report, ICE was provided with the following updated documents related to property development:

- Core Design, November 2013, *Short Plat Layout Exhibit, Inneswood Estates*, sheet 1, scale 1 inch = 60 feet.
- Core Design, November 2013, *Roadway Exhibit, Inneswood Estates*, sheet 1, scale 1 inch = 30 feet.
- Core Design, October 2013*, Preliminary Site Plan, Spak Property*, scale 1 inch = 100 feet.

The triangular-shaped property occupies about 10½ acres located generally west of the intersection of Newport Way NW and NW Juniper Street as shown on Figure 2. The conceptual development plan for this property involves developing the lower, central portion of the property into a multi-family apartment building (referred to in this report as the "Apartment Building Area") and subdividing the upper portion of the property into nine residential lots (referred to this report as the "Residential Building Area" and in the project documents as "Inneswood Estates"). A new road will be created along a portion of the west property line to access the Residential Building Area. A third area, referred to on the project plans as a "Future Development Area" is located in the southeast corner of the property.

The hillside that separates the Apartment Building Area from the Residential Building Area contains Steep Slope Hazard Environmentally Critical Areas (ECAs) according to the City of Issaquah Land Use Code (LUC), Chapter 18.

2.2 APARTMENT BUILDING AREA

The apartment building is proposed as a four-story, wood-frame building with basement parking. The conceptual grading plan for access and the building may require cuts up to 15-feet deep for the west (uphill) basement wall along other low walls (less than 5-feet high), possibly rockeries, where the property parallels Newport Way NW to provide for grade changes. Paved access and at-grade parking will be constructed north, east and south of the apartment building. An underground stormwater vault is may be located in the paved access/parking area north of the apartment building in the area paralleling Newport Way NW. We understand that the City of Issaquah may reconfigure the intersection of NW Juniper Street and Newport Way NW into a traffic circle, though the plans for this project are not available at this time.

2.3 RESIDENTIAL BUILDING AREA

The residential houses are proposed as two-story, wood-frame structures, some with a daylight basement depending on the lot grades. Minimal grading is planned, though cuts up to 8-feet deep may be required to accommodate the daylight basement construction. Access to the residential lots will consist of a private road extension off of the east end of NW Inneswood Place. Stormwater from the houses (roof downspout, footing drains and driveway drains) will be routed by tightline to the proposed stormwater vault in the Apartment Building Area. The stormwater tightline will be routed down the hillside area that separates the Apartment Building Area and the Residential Building Area.

3.0 CITY OF ISSAQUAH CRITICAL AREAS CONSIDERATIONS

As proposed, the apartment and house building construction will require a reduction of Steep Slope Hazard Area (slopes more than 40 percent grade) buffer. The areas along the hillside with slopes at a 40 percent grade or greater and more than 20-feet high have been mapped by Core Design as shown in Figure 2.

A Steep Slope Hazard Area is defined by the City of Issaquah (LUC Chapter 18.10.390) as *"any ground that rises at an inclination of forty (40) percent or more with a vertical elevation change of at least ten (10) feet or more for every twenty-five (25) feet of horizontal distance."* Development in the vicinity of a steep slope requires a "building setback" of 15 feet (LUC Chapter 18.10.580.B) and a "buffer" (an area of undisturbed vegetation) of 50 feet (LUC Chapter 18.10.580.A.1). The buffer may be reduced to a minimum of 10 feet pursuant to a Critical Area Study provided that "*the reduction will not reduce the level of protection to the proposed development and the critical area"* (LUC Chapter 18.10.580.A.2).

Exemptions to the Steep Slope Hazard regulation include, *"Slopes forty (40) percent and steeper with a vertical elevation change of up to twenty (20) feet may be exempted from the provisions of this section (through Level 1 Review or through the appropriate land use permitting process), based on the City review and acceptance of a soils report prepared by a geologist or licensed geotechnical engineer when no adverse impact will result from the exemption"* (LUC Chapter 18.10.580.E1), and *"Any slope which has been created through previous, legal grading activities may be regarded as part of an approved development proposal. Any slope which remains equal to or in excess of forty (40) percent following site development shall be subject to the protection mechanisms for steep slopes"* (LUC Chapter 18.10.580.E2).

4.0 SCOPE OF SERVICES

The purpose of our services was to explore subsurface soil and ground water conditions at the property as a basis for developing preliminary geotechnical recommendations for site development. Specifically, our services included the following:

- Review readily available geologic and geotechnical information regarding conditions of the project area.
- Complete a geologic reconnaissance of the property and adjacent areas to evaluate current site conditions, with emphasis on the slope area.
- Explore subsurface soil and ground water conditions by excavating twelve test pits (six test pits in the Apartment Building Area and six test pits in the Residential Building Area) using a track-mounted excavator provided by Vineyards Construction, LLC.
- Provide a quantified evaluation of the stability of the steep slopes using the computer application DLISA.
- Provide recommendations for buffer and building setback criteria from Steep Slope Hazard Areas.
- Provide preliminary recommendations for building foundation support and site drainage related to construction near the Steep Slope Hazard Areas.

- Provide preliminary recommendations for earthwork including stripping and excavation of unsuitable soils, fill compaction and subgrade preparation requirements, and suitability of on-site soils for use in structural fills. This included evaluation of the effects of weather and/or construction equipment on the workability of site soils.
- Provide recommendations for underground utility installation including backfill materials, dewatering and shoring requirements, and provide recommendations for excavation and trench side slopes and placement and compaction of bedding and backfill materials.
- Provide recommendations for cut and fill slopes.
- Provide recommendations for foundation support for the proposed buildings including allowable bearing pressures, settlement estimates and support of on-grade floor slabs.
- Provide recommendations for slope support structures, including soldier pile walls and conventional concrete walls.
- Provide recommendations for rockery design and construction, and other slope support options.
- Provide preliminary recommendations for parking and access subgrade preparation.
- Provide design values for foundation support, uplift, friction, lateral soil pressures, and estimated postconstruction settlement performance of the stormwater vault.
- Provide recommendations for temporary and permanent drainage and erosion control measures.

5.0 GEOLOGIC SETTING

Regional geologic mapping of this area by the US Geological Survey (USGS, Booth, D.B., et al., 2006, *Geologic Map of the Issaquah 7.5' Quadrangle, King County, Washington*) shows that the hillside and upland area of the property are underlain by Pre-Olympia Age Glacial Deposits. Pre-Olympia Age Glacial Deposits are described by the USGS (2006) as *weakly to strongly oxidized silt, sand, gravel and local till of glacial origin*. A thin (a few feet thick) layer of Ice-Contact deposits is mapped by the USGS (2006) to overlie the Pre-Olympia Age Glacial Deposits in the upland area along the southwest boundary of the property. Ice-Contact Deposits are described as soils with a high *percentage of silt intermixed with granular sediments, resulting in poorly sorted stratified sediment and local diamicts.*

The nearly-level area at the base of the hillside is mapped by the USGS (2006) as being underlain by Alluvium and Peat Deposits. Alluvium is described as *moderately sorted cobble gravel, pebbly sand, and sandy silt mapped along major stream channels*. Peat Deposits are described as *peat, accumulated in bodies greater than about 1 m in thickness*.

Though not mapped by the USGS, Colluvium and Weathered Soil typically mantle the ground surface as a result of weathering (slopewash, wetting/drying, freeze/thaw, root and animal bioturbation) of the underlying native geologic units.

6.0 SITE CONDITIONS

6.1 SURFACE CONDITIONS

Brian Beaman and Jeff Schwartz of ICE completed a geologic reconnaissance of the project site on March 15, 2012. The weather during the site visit was overcast with rain and temperature in the 40s.

The property is situated along an east-northeast-facing hillside adjacent to the Issaquah Creek Valley. The property is currently undeveloped and forested with exception of a single-family house centrally-located along Newport Way NW (905 Newport Way NW). The property is bordered by residential properties to the south and west and Newport Way NW and commercial properties, including the King County Library System administration building to the east.

From the east-central portion of the property where the apartment building is proposed, the ground surface is nearly level at about Elevation 75 feet, then ascends a hillside at about a 30 to 50 percent grade then becomes less steep at about Elevation 180 to 200 feet where the upland (Residential Building Area) is encountered. The upland is a gently undulating surface with a slight gradient up to the southwest at less than 10 percent grade to the southwest corner of the property at about Elevation 220 feet. Slope areas that are inclined greater than 40 percent grade and more than 20-feet high (referred to as Regulated Steep Slope Hazard Areas) are shown on the Steep Slope Hazards Map, Figure 3. Also shown on Figure 3 are slope areas that are greater than 40 percent grade and, in part, were created by previous legal grading (referred to as Exempt Steep Slope Hazard Areas).

The existing house on the property along Newport Way NW was constructed in 1929 (according to King County Department of Assessments). An unused chicken coop and smokehouse are located north of the house. We observed segments of an abandoned road grade that is aligned along the base of the slope north and south of the existing house. The nearly-level area between the base of the slope and the east property boundary is predominately lawn with a few scattered fruit trees. Based on information provided by Stephen Spak, the property owner at the time of our geologic reconnaissance, a road was formerly aligned within the yard area. Mr. Spak indicated that the former road was covered over with soil which allowed the lawn to grow in this area.

The upland and hillside area is forested with predominately large Douglas fir, cedar and deciduous trees. We observed the hillside area to consist of generally planar-surfaced slopes with a broad (100- to 200-foot wide) swale extending the full height of the hillside roughly centered on the existing house (and proposed apartment building). In general, we observed straight-trunked conifer trees and scattered old-growth stumps across the hillside. Within the swale we observed primarily large deciduous trees.

We did not observe surface evidence of landsliding or slope instability during our reconnaissance, including bare soil scarps, hummocky topography or groups of leaning or toppled trees. We observed water flowing at about 10 gallons-per-minute from a concrete wall drain along the west end of the existing house. According to Mr. Spak, this drain was installed to intercept ground water seepage that occurs behind the concrete wall and typically flows year-round though the flow diminishes in the summer and early fall months.

No surface water was observed on the property at the time of our site visit.

6.2 SUBSURFACE CONDITIONS

6.2.1 General

Subsurface conditions across the site were explored by excavating twelve test pits (Test Pits TP-1 through TP-12) to depths ranging from about 8 to 15½ feet on March 22 and 23, 2012 at the locations shown on Figure 2. Details of the subsurface exploration program, along with the test pit logs, are presented in Appendix A.

The observed soil conditions in the test pit explorations are generally consistent with the previously described regional geologic mapping. None of the proposed development is within areas underlain by Alluvium or Peat. Ice-Contact Deposits were not encountered in the test pits, but could be present in the upland area.

6.2.2 Apartment Building Area (Test Pits TP-1 through TP-6)

Test Pits TP-1 and TP-3 were excavated within the nearly level lawn area adjacent to Newport Way NW. The test pit explorations encountered about 3 to 4 inches of Sod and Topsoil underlain by approximately

½ to 2½ feet of Fill consisting of very loose to medium dense silty gravel with sand and silty sand with gravel and cobbles. Abundant roots and charcoal fragments were observed in the fill in Test Pit TP-3. The Fill was underlain by Colluvium and/or Weathered Soil to a depth of about 4 to 4½ feet consisting of medium dense silty sand with variable amounts of gravel and cobbles. Pre-Olympia Age Glacial Deposits underlie the Colluvium and Weathered Soil consisting of dense sand with variable amounts of silt and gravel, and gravel with silt, sand and cobbles to the completion depths of the Test Pits TP-1 and TP-3 at about 8 to 9 feet.
Test Pits TP-2, TP-4, TP-5 and TP-6 were excavated along the base of the hillside area, generally in the area of the basement wall for the proposed apartment building. The test pit explorations encountered about ½ to 1 foot of Sod or Forest Duff and Topsoil underlain by about 2 to 6½ feet of Colluvium and Weathered Soil consisting of very loose to dense sand and gravel with variable amounts of silt, cobbles boulders and roots. Pre-Olympia Age Glacial Deposits were encountered under the Colluvium and Weathered Soil to the full depth of the Test Pits TP-2, TP-4, TP-5 and TP-6 at about 8½ to 15½ feet; these deposits consisted of dense to very dense sand and gravel with variable amounts of silt, cobbles and boulders, with occasional interbeds of very stiff to hard silt.

As previously described in section **5.0 GEOLOGIC SETTING**, peat (a highly compressible organic soil) has been regionally mapped by the USGS (2006) in the general property area. Based on our general knowledge of the site location and geologic conditions, we do not expect that peat underlies the property, though it could be present along the east property line, primarily along Newport Way NW and areas to the north and east.

Ground water seepage was observed in Test Pit TP-1 (moderate seepage) and Test Pit TP-3 (rapid seepage) at a depth of about 6 feet. Slow ground water seepage was observed in Test Pit TP-4 from a depth of about 13 and 15 feet.

6.2.3 Residential Building Area (Test Pits TP-7 through TP-12)

Test Pits TP-7 through TP-12 completed in the Residential Building Area encountered about ½ to 1 foot of Forest Duff and Topsoil underlain by about 2½ to 5½ feet of Weathered Soil consisting of soft to stiff silt with variable amounts of clay and sand and very loose to dense sand with variable amounts of silt and gravel. The Weathered Soil was underlain by between Pre-Olympia Age Glacial Deposits consisting of very stiff to hard silt with variable amounts of sand and dense to very dense sand with variable amounts of silt and gravel to the completion depths of about 9 to 10 feet.

Slow to moderate ground water seepage was observed in Test Pits TP-7, TP-8 and TP-9 between depths of about 3 and 4 feet.

6.2.4 Ground Water Conditions

Our interpretation of ground water conditions is based on the geologic conditions, our site observations, the results of our test pit explorations and our general knowledge of hydrogeologic conditions in the property area. We expect that ground water occurs as a locally perched (shallow – less than 5-feet deep), seasonal ground water system within the relatively permeable Weathered Soil layer (the Weathered Soil layer is typically looser than the underlying native soil) that mantles the upland (Residential Building Area) and hillside area. This shallow ground water system accumulates during the wet season along the underlying geologic contact with the less permeable Pre-Olympia Age Glacial Deposits. As previously described, Test Pits TP-7, TP-8 and TP-9 encountered this shallow ground water. It is likely that this shallow perched ground water dries out by late summer.

We also expect a deep (more than 5-feet deep and likely much deeper) ground water system to exist within the Pre-Olympia Age Glacial Deposits. This ground water system likely occurs within the more permeable layers (sand and gravel) that are present in Pre-Olympia Age Glacial Deposits and likely contains multiple layers of ground water year-round. Because the Pre-Olympia Age Glacial Deposits are "stratified" (near-horizontally layered), these ground water "layers" are truncated by the hillside that separates the Apartment Building Area and the Residential Building Area. Ground water seepage and springs can occur where these layers are truncated by the hillside, though surface evidence of these springs and seepage areas was not observed on the hillside during our site reconnaissance. This is likely because the overlying (looser) Weathered Soil and Colluvium are permeable enough to transmit this ground water as shallow to moderately deep (less than 15-feet deep) subsurface flow down the hillside. Evidence of this shallow subsurface flow is exhibited by the ground water that is intercepted by the existing house wall drain and Test Pits TP-1, TP-3 and TP-4.

6.2.5 Slope Stability Analysis

Slope stability analysis requires accurate surface topographic information and an appropriate amount of subsurface data for which soil and ground water conditions can be confidently interpreted. The topographic information used for our analysis is based on a field survey by Core Design (November 2013). Subsurface data used for our slope stability analysis included test pit explorations (TP-1 through TP-12) and our general knowledge of the area including a review of test pit and test borings in the general area (obtained from the Washington State Department of Natural Resources, Washington State Geologic Information Portal).

Based on our site observations and experience with similar topographic, geologic and hydrogeologic conditions, a deep-seated landslide (circular failure plane more than 10 feet below the ground surface) is a low risk for the existing or developed conditions, provided the recommendations for re-support of cut slopes (structural walls), and buffers/drainage are implemented for the Apartment Building Area and the Residential Building Area.

For the purpose of evaluating shallow landsliding, we developed a model of the landslide condition where a slope failure involves the weathered soil layer (5-feet thick). The location of the critical section for this purpose is in the steepest section of hillside area as shown on Figure 2. The failure occurs along a planar surface parallel to the slope, with ground water conditions ranging from 2, 3 and 4 feet (Case 1, 2 and 3, respectively) below the ground surface. We also included the effects of "tree surcharge" (a driving force) and "apparent cohesion" (a resisting force) that is provided by tree roots. We assigned the weathered soil an angle of internal friction of 30 degrees, zero cohesion and a moist unit of 120 pounds per cubic foot.

Slope stability analyses were conducted for each case using the computer program DLISA (Deterministic Level I Stability Analysis) that was obtained from the US Forest Service (Rocky Mountain Research Center, Forestry Sciences Laboratory, Moscow, Idaho). This program has the ability to analyze slope stability for a wide range of failure geometries and variable subsurface soil and ground water conditions including the ability to allow for the input of tree surcharge and apparent root cohesion for complex evaluations of vegetated steep slope areas.

In stability analyses, the relative stability of a slope is expressed in terms of a factor of safety (FOS) against sliding for the most likely potential failure surface. A FOS of 1.0 corresponds to the conditions in which the resisting and the driving forces are equal, and failure would theoretically be imminent as the result of a decrease in the resisting force or an increase in the driving force. A FOS greater than 1.0

indicates that the forces tending to resist sliding are greater than the forces tending to cause sliding. Typically, a FOS between 1.3 and 1.5 would be considered adequate for static conditions.

The following is a summary of the FOS for each case (varying ground water condition):

Case 1 (shallow ground water)	FOS 1.9
Case 2 (intermediate depth ground water)	FOS 2.0
Case 3 (deep ground water)	FOS 2.1

Under seismic loading, we would expect each of these FOS's to decrease by about 0.5.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

It is our opinion that the proposed apartment and residential building and related access/stormwater vault facilities may be constructed generally as planned. The buildings may be adequately supported using conventional shallow reinforced concrete spread footings that are founded on medium dense or denser native soil, or on a pad of structural fill that extends to the medium dense or denser native soil or the medium dense Weathered Soil provided that no roots larger than 1 inch in diameter are present. Buildings constructed adjacent to steep slopes must be setback (buffered) from the top or toe of the steep slopes as described subsequently in this report. Site drainage is likely the most critical factor for site development in terms of construction and long-term satisfactory performance not only for the structures, but for the intervening undeveloped hillside that separates the two proposed development areas along with re-support of cut slopes with structural walls.

7.2 ENVIRONMENTALLY CRITICAL AREAS

7.2.1 General

ICE reviewed the following Environmentally Critical Areas (ECA's) maps and definitions for the City of Issaquah including Erosion, Landslide, Seismic, Steep Slope and Coal Mine Hazards and Aquifer Recharge areas. According to the maps, Steep Slope Hazard Areas are present at the property.

7.2.2 Steep Slope Hazard Areas

Based on our site observations and review of current site plans, local areas of Steep Slope Hazard Areas (slopes that are inclined more than 40 percent grade with a vertical height of at least 20 feet) are shown on Figure 3. We also observed local areas of slopes that are inclined more than 40 percent grade and created, in part, by previous legal grading activities as shown on Figure 3. We did not observe surficial evidence of active landsliding in these Steep Slope Hazard Areas, or the less steep areas that comprise the entire hillside area. We were initially concerned about the broad swale that occurs immediately upslope from the proposed apartment building. However, Test Pit TP-4, excavated directly in the center and near that base of this swale (a proposed apartment building wall location), encountered competent soil conditions (Pre-Olympia Age Glacial Deposits) at a relatively shallow depth (about 5½-feet deep).

As previously described, the City of Issaquah requires that foundation elements of buildings, including retaining walls, decks and other appurtenant structures, be set back from the top or toe of Steep Slope Hazard Areas. The degree to which this regulation is applicable is a function of the overall height of the steep slope (40 percent or more) area. Steep Slope Hazard Areas (slopes that are 40 percent or greater and more than 20 feet in height) should be provided with a 10-foot buffer (reduced from the standard buffer of 50 feet) and a 15-foot building setback (total buffer and building setback of 25 feet).

The City of Issaquah LUC allows for slopes that are 40 percent or greater and less than 20-feet high, or that have been created by previous legal site grading, to be exempt by the geotechnical engineer (subject of this report), based on the City of Issaquah's review and acceptance. In our opinion, slopes that are 40 percent or greater and less than 20-feet high should be exempt from the Steep Slope Hazard Area Development Standards (buffer and setback) based on our knowledge of the site conditions (LUC Chapter 18.10.580.E1). This includes the lower ridgeline area where the steep slope is less than 20-feet high which is part of an overall much larger steep slope area and is topographically in a more favorable setting. This specific area (shown on Figure 3 shaded in green) forms along the toe of a distinct ridgeline with limited overall slope height, is planar surfaced, dry and forested with mature trees. Some of these steep slopes that are less than 20 feet in height (most are actually less than 10 feet in height) and occur along the frontage with Newport Way NW and in the vicinity of Test Pits TP-1 and TP-2 (shown in black cross-hatch on Figure 3) are the result of previous legal grading activities, and would be exempt for this reason (LUC Chapter 18.10.580.E2).

Our recommendations for buffer and building setback widths are contingent on protecting and maintaining the steep slope vegetation during and after construction, and implementing the recommendations included in this report.

As previously described in section **6.2.5 Slope Stability Analysis**, the stability of the hillside area is adequate based on our analysis (FOS ranging from 1.9 to 2.2) for static conditions. The majority of this hillside area will remain forested (undeveloped). However, the proposed development, though adequately buffered from Steep Slope Hazard Areas, should require implementation of the recommendations in this report for re-support of cut slopes and surface and subsurface drainage measures.

7.3 SITE PREPARATION

Temporary erosion control measures such as silt fences, straw bales, and detention structures should be installed to local standards for erosion and sediment control prior to the start of construction.

We recommend that the building sites, road and parking areas be stripped of vegetation and significant organic material and that this material be removed from the site. Tree stumps and roots over 2 inches in diameter should be grubbed and removed from these areas. The existing house and foundation elements, along with the abandoned underground utilities (water line, septic tank and drainfield) and the former road bed should be removed.

During dry weather conditions, the depth of stripping is expected to typically range from 1 to 1½ feet unless excessive disturbance is caused by the clearing operations. Stripping to a greater depth should be expected near grubbed tree stumps, abandoned utilities or if the stripping is completed during wet weather.

After stripping is complete, the exposed surface should be proofrolled or probed. Proofrolling should be accomplished using heavy construction equipment such as a fully-loaded dump truck. The site should be proofrolled only during dry weather. Probing should be used to evaluate subgrade conditions during periods of wet weather. Soft areas noted during proofrolling or probing of the ground surface should be overexcavated and replaced with compacted structural fill, as outlined in the following section.

The on-site soils contain a sufficient amount of fines (material passing the US No. 200 sieve) to be moisture-sensitive. These soils may be difficult to work on and difficult or impossible to compact during periods of wet weather. Therefore, we recommend site earthwork be completed during periods of dry weather.

7.4 STRUCTURAL FILL

New fill in building and road areas should be placed as compacted structural fill. Structural fill should be free of debris, organic contaminants and cobbles or rock fragments larger than 6 inches. The suitability of soil for use as structural fill will depend on its gradation and moisture content. As the amount of fines increases, soil becomes increasingly more sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. The on-site soils contain sufficient fines to be moisture-sensitive. During dry weather, the on-site soils can be used as structural fill, provided that these materials are conditioned to the proper moisture content for compaction.

The on-site soils will not be suitable for use as structural fill during wet weather. On-site soils considered unsuitable for use as structural fill during any weather conditions include the sod, and Colluvium/Weathered Soil where dense roots exist unless the roots can be satisfactorily segregated from the soil matrix. If structural fill must be placed during wet weather, we recommend the use of imported sand and gravel containing less than 5 percent fines by weight relative to the fraction of the material passing the ¾-inch sieve. The imported sand and gravel should be moisture-conditioned as necessary for proper compaction.

Structural fill should be mechanically compacted to a firm, nonyielding condition. Structural fill in the building areas should be compacted to at least 95 percent of the maximum dry density (MDD) in accordance with ASTM Test Method D 1557. Access road and parking fill areas, including utility trench backfill, should be compacted to at least 95 percent of the MDD. Structural fill should be placed in loose lifts so that adequate compaction can be achieved throughout the thickness of the lift. The thickness of the lift will depend on the soil type and gradation, the type of compaction equipment and other factors and should be evaluated during construction. Each lift should be conditioned to the proper moisture content and compacted to the specified density.

We recommend that a representative from ICE be present during site stripping, proofrolling and/or probing of the subgrade and structural fill placement operations. Our representative will observe the stripping, evaluate subgrade performance, complete in-place moisture-density tests to evaluate compliance with the compaction specifications, and advise the contractor on modifications to procedures which may be appropriate for the prevailing conditions.

7.5 UNDERGROUND UTILITY CONSIDERATIONS

Based on our explorations, most underground utilities less than 10-feet deep will encounter Colluvium/Weathered Soil or Pre-Olympia Age Glacial Deposits. In our opinion, normal bedding requirements as specified by the pipe manufacturer should be satisfactory. Utility backfill in the roadway or in other settlement-sensitive areas should be compacted to 95 percent of the MDD. Ground water seepage may be encountered, especially if trenching is conducted in the wet season. We expect that dewatering the trench with pump in a sump should be adequate to remove ground water.

We expect that most of the Colluvium/Weathered Soil and Pre-Olympia Age Glacial Deposits that are excavated may be reused as trench backfill during periods of extended dry weather provided these soils are moisture-conditioned for proper compaction, and that roots have been removed as previously described.

7.6 SLOPES

7.6.1 General

Abrupt transitions between cut and fill road sections may result in damage to paved roads. Therefore, we recommend that all cut/fill transition zones be graded such that the fill thickness does not increase by more than 2 feet within a 10-foot horizontal distance within 20 feet of the cut/fill transition line.

Unprotected cut and fill slopes will be susceptible to erosion until a good protective cover is established. Temporary (or permanent) slope erosion protection should be established as earthwork progresses. Permanent slope protection can be achieved by an adequate grass or vegetative cover.

7.6.2 Fill Slopes

We recommend that fill slopes founded on medium dense or denser native soils be sloped at 2H:1V (horizontal to vertical) or flatter. Slopes of 3H:1V or 4H:1V should be used where practical to provide for ease in landscaping and maintenance. Ground surfaces that will receive fill should be properly stripped of vegetation and organic matter before placing fill. Fill placed on existing slopes that are steeper than 4H:1V should be properly keyed into the native slope surface. This can be done by constructing the fill in a series of 4- to 8-foot-wide horizontal benches cut into the slope. The fill should be placed in horizontal lifts.

7.6.3 Cut Slopes

Temporary cuts less than 4 feet in height may be made near-vertical in competent soil. Temporary cuts greater than 4 feet in height in the Pre-Olympia Age Glacial Deposits may be made at 1H:1V or flatter. Temporary cut slopes in Colluvium/Weathered Soil should be no steeper than 1.5H:1V. Permanent cut slopes in the Colluvium/Weathered Soil, Pre-Olympia Age Glacial Deposits or structural fill should be inclined no steeper than 2H:1V.

Where localized seeps are encountered on a cut slope, we recommend placing quarry spalls to control erosion on the face of the cut slope. It may also be necessary to install a collector and tightline the water to the storm drainage system.

7.7 FOUNDATION SUPPORT

We recommend that the buildings be supported on conventional spread footings founded on medium dense or denser native soils, or on a pad of structural fill that extends to the medium dense or denser native soils. If used, the zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to its thickness in all directions. The structural fill should be compacted to at least 95 percent of the MDD. Footing excavations should be observed by a representative from our firm before placing structural fill or pouring concrete to evaluate whether suitable bearing soils have been exposed.

The depth of embedment below the lowest adjacent finished grade for exterior and interior footings should be at least 18 inches and 12 inches, respectively. We recommend a minimum footing width of at least 15 inches for continuous wall footings and 18 inches for isolated column footings. For footings designed and constructed according to the above criteria, we recommend an allowable bearing pressure of 2,500 pounds per square foot (psf). This value applies to the total of all dead plus long-term live loads, exclusive of the weight of the footing and overlying backfill. This value may be increased by one-third when considering wind or seismic loads.

Although the native soils are normally relatively strong in a fresh and undisturbed condition, the Pre-Olympia Age Glacial Deposits are moisture-sensitive and can be expected to soften easily when exposed to moisture and foot or machinery traffic. During wet weather, it may be necessary to protect the footing excavations against disturbance by using a thin layer of crushed rock or lean mix concrete.

The total settlement of footings supported on medium dense to very dense native soil, or on structural fill placed and compacted in accordance with our recommendations, is expected to be less than 1 inch. Differential settlements over a distance of 25 feet are expected to be less than ½ inch. Settlements will occur rapidly as loads are applied and postconstruction settlements should be minor provided that the above recommendations for foundations are applied.

7.8 FLOOR SLAB SUPPORT

Floor slabs can be supported on-grade provided that slab subgrade areas are prepared as recommended above under sections **7.3 SITE PREPARATION** and **7.4 STRUCTURAL FILL**, and that the following capillary break measures are installed.

We recommend that a compacted base course layer consisting of at least 4 inches of gravel containing less than 3 percent fines be placed on the subgrade to provide uniform support and a capillary break beneath the slab. A vapor retarder should be placed beneath the slab if moisture control in the slab is critical (i.e., where tile or carpeting is to be glued to the slab). This vapor retarder should consist of polyethylene sheeting. A layer of clean sand not more than 2 inches in thickness may be placed over the polyethylene sheeting. The vapor retarder should be placed immediately below the slab.

We estimate that the settlement of floor slabs due to uniform areal loads of on the order of 150 psf will be less than ½ inch. These settlements are expected to occur rapidly upon load application.

7.9 SUBGRADE (BASEMENT) WALLS

7.9.1 General

We expect that the basement walls for the houses in the Residential Building Area will be designed and constructed as a conventional reinforced concrete retaining (subgrade) walls.

The basement wall for the apartment building also could be constructed in this manner, but at some risk of slope instability. For this reason, we are providing recommendations for a pre-installed wall, referred to as a soldier pile wall, which allows for the basement wall to be installed prior to excavation, then the soil downslope of the wall can be removed without risk of slope instability. The primary risk is related to encountering a large amount of ground water seepage should an open cut be used. The zone(s) of ground water within the Pre-Olympia Age Glacial Deposits is difficult to predict because of the layered character of these deposits. To reduce the risk of encountering zones of ground water seepage (but not eliminate the risk), we recommend that this wall construction be completed during the dry season, such as between July and September. Alternatively, a combination of wall types could be considered. Where wall heights are less than 6 feet, then a conventional reinforced concrete wall could be used, with the soldier pile wall being used where wall heights exceed 6 feet.

7.9.2 Reinforced Concrete Retaining Wall

Lateral Pressures - The lateral soil pressures acting on retaining walls and basement walls depend on the nature and density of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed. For walls that are free to yield at the top at least one one-thousandth of the height of the wall, the lateral static soil pressure will be the active case. If the wall is braced or otherwise restricted from yielding, at-rest lateral pressures will develop. For the walls which are backfilled and drained as described in the following paragraphs, the design active or at-rest lateral pressures, expressed as an equivalent fluid density, are presented in the following table:

Design Lateral Wall Pressures

Back Slope	Active Case	At-Rest Case
Horizontal	35 pcf	50 pcf
2H:1V	50 pcf	60 pcf

pcf = pounds per cubic foot

A drainage layer of clean gravel ("clean gravel" in this case is defined as "clean, washed, rounded gravel with a uniform diameter between ¾ and 1¼ inches) should be placed behind the wall and should extend at least 18 inches horizontally from the back of the wall. Backfill behind the wall should be compacted to at least 92 percent of the MDD as determined by ASTM D 1557 Test Method. Overcompaction should be avoided. A drain consisting of 4-inch diameter, smooth-walled perforated PVC pipe, should be placed at the bottom of all retaining walls. The drainage zone should include a non-woven geotextile fabric, such as Mirafi 140N at the base and between the drainage zone and other wall backfill. We recommend that the walls be waterproofed if the potential for moisture on the inside of the walls is not acceptable.

The above-recommended lateral soil pressures do not include the effects of surcharges such as floor loads, traffic loads or other surface loading. The effective wall height used to calculate lateral pressures should be increased by 1 foot for each 100 psf surcharge load.

Lateral Resistance - The soil resistance available to resist lateral foundation loads is a function of the frictional resistance which can develop on the footing base and the passive resistance which can develop on the footing sides as the structure tends to move into the soil. For footings founded on native soils in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to the dead load forces. The allowable passive resistance on the sides of footings cast neat against undisturbed native soils or structural fill may be computed using an equivalent fluid density of 300 pcf. The above coefficient of friction and passive equivalent fluid density values both include a factor of safety of about 1.5.

7.9.3 Soldier Pile Wall

Active Lateral Soil Pressures – We expect that the soldier pile wall will retain soil with a backslope of about 2H:1V. Active lateral soil pressures acting on the wall can be computed using 55 pcf equivalent fluid density (triangular distribution).

The lateral soil pressures should be assumed to act over the center-to-center spacing between the soldier piles above the passive soil zone and over the soldier pile diameter below this zone. The active lateral soil pressure includes a factor of safety of 1.5.

Passive Lateral Soil Pressures - Passive lateral soil pressures acting on the wall can be computed using 300 pcf equivalent fluid density (triangular distribution) beginning at the base of the basement floor slab section. Passive lateral soil pressures should be assumed to act over three times the pile diameter, or over the soldier pile center-to-center spacing, whichever is less. The passive lateral soil pressure includes a factor of safety of 1.5.

Drainage and Lagging - A suitable drainage system should be installed behind the wall to prevent the buildup of excessive hydrostatic ground water pressures behind the wall.

Permanent lagging should be installed between the soldier piles to retain the soil. Wherever possible, the lagging should extend at least to the base of the basement floor slab elevation. Because this wall will be used as a finished basement wall, the structural engineer should review the method of lagging to accomplish this purpose.

Construction Considerations (Installation and Cut Slopes) – Dense native soils, cobbles or boulders may be encountered while drilling the soldier pile shafts. The contractor should be prepared to utilize drilling methods which can penetrate through these materials where encountered.

Some of the surficial soils are in a loose condition and may contain perched ground water or deeper ground water zones within the Pre-Olympia Age Glacial Deposits. This loose and/or wet material could tend to cave into the shaft excavation. The contractor should be prepared to complete the shaft excavation in such a way that caving is prevented (e.g., casing).

Temporary slopes may be necessary during installation of lagging. Temporary cut slopes of 1.5H:IV or flatter may be used provided that no significant ground water seepage is encountered. Flatter cut slopes are recommended when significant seepage is encountered or if caving is persistent. In any case, it is the sole responsibility of the contractor to follow WISHA (Washington State Industrial Safety and Health Act) regulations for excavations and shoring.

7.10 SLOPE SUPPORT OPTIONS

7.10.1 Rockery Walls

General - Rockery walls may be used in both cut and fill areas. In cut areas where medium dense (if the cut slope will temporarily stand vertically), or denser native soils are exposed, rockery walls may be up to 8-feet high if the ground surface behind the wall is level, or up to 6-feet high for a 2H:1V backslope. In fill areas, we recommend limiting the wall height to 4 feet when retaining fill having a level backslope, or 2 feet for a 2H:1V backslope.

We recommend that rockery walls be founded on medium dense or denser native soils or on structural fill placed to the recommended standard. In our opinion, when adequate foundation support and suitable materials are present such as those outlined above, rockery walls will provide the most cost-effective earth retention system. Specific construction guidelines for rockery walls are presented in the following paragraphs.

Construction Guidelines - The primary purpose of a rockery is to protect the slope face from erosion and raveling, while providing limited soil retention. The base of the rockery should be embedded at least one-half the thickness of the lowest course of rocks or 18 inches below the adjacent ground surface, whichever is greater. The rockery should be supported on medium dense or denser, undisturbed native soils or on compacted structural fill. The final rock wall face should be constructed with a batter of between 1H:5V and 1H:6V.

The rockery rocks should be tabular and rectangular. Rocks should be hard, sound, durable and free of weathered portions, seams, cracks and other defects. Based on the height of the cut and the slope behind the rockery, we recommend that the rockeries be constructed using rock weights from about 750 to 5,000 pounds (3- to 5-man rocks as defined by Associated Rockery Contractors). The rock density should not be less than 160 pcf. The lower 4 to 6 feet of the rockery should be constructed using 4- to 5-man size rocks.

Rock selection and placement should be accomplished to reduce the number and size of voids. In the exposed face of the wall, no openings greater than 6 inches in dimension in any direction should be permitted. Rock courses should be gradational in size from bottom to top with the largest rocks of uniform size being placed for the lowest two courses. The contact between rocks should slope downward to the back side of the rockery. Each course of rocks should be seated tightly and evenly on the course beneath. After seating each course of rock, voids between the rocks should be chinked on the back with quarry spalls to eliminate passage of backfill material. Backfill immediately behind the rockery should consist of quarry spalls. The spalls should consist of well-graded ¾- to 4-inch crushed rock and should be durable, uncontaminated by soil or other debris, and not readily susceptible to weathering.

The quarry spall fill should be placed to a width of not less than 18 inches between the rockery and the face of the cut. The spalls should be placed and compacted in lifts to a level approximately 2 inches below the top of each course of rocks as they are placed, until the uppermost course is placed. Any backfill material falling onto the bearing surface of one rock course must be removed before setting the next course. Rock placement should be such that each rock above the base course will be supported on two rocks in the next lower row.

A perforated drainpipe should be embedded in the backfill at the base of the rockery. This drain should discharge to the storm drain system or daylight at a location which will not impact the adjacent road or other moisture-sensitive areas.

Rockeries should be installed by a qualified contractor experienced in rockery construction. The construction should be observed by a representative of our firm.

7.10.2 Crib Walls

Interlocking crib walls could be considered to retain any of the cut slopes or fill embankments currently planned in the project area. This type of retention system consists of elements, typically made of precast concrete or metal, that interlock to form a box. Clean granular backfill material is placed and compacted inside the box. Horizontal loads from the slope being retained are resisted primarily by gravity. This type of retaining wall can withstand some settlement. The amount of settlement depends on foundation support conditions and construction techniques.

Significant excavations are required for the construction of crib walls in cut areas. The width of the excavation is typically on the order of 1/2 to 2/3 of the height of the wall. We expect that the temporary excavation will be sloped at about 0.5H:1V where the cut is made in the native Pre-Olympia Age Glacial Deposits and 1H:1V where the cut is made in Colluvium/Weathered Soils.

7.10.3 Mechanically Stabilized Earth (MSE) Walls

MSE walls are an economical means of retaining fills up to 20 feet or more high. They are generally not cost-effective for cuts where soil must be excavated to allow placement of the reinforcement strips (geogrids). Geogrids are embedded in the compacted fill as the level of the fill is raised. The geogrids effectively stabilize a large mass of soil by developing friction between the grids and the soil and increasing the shear strength of the mass. The face of the reinforced fill may be nearly vertical if it is covered with architectural blocks which are integral with the grids. If the face of the reinforced mass is left unfinished, it can be inclined more steeply than an unreinforced fill slope.

Well-drained granular soils are best suited for use as fill when constructing a MSE wall system. The on-site soils contain a relatively high percentage of fines and will not be suitable for use in the reinforced fill zone.

For design of the MSE walls, we recommend using a strength angle (phi) of 32 degrees for medium dense or denser native undisturbed soils/Colluvium and Weathered Soil, or structural fill compacted to at least 95 percent of the MDD. A drainage layer of crushed rock with less than 3 percent fines should be placed behind the wall and should extend at least 18 inches horizontally from the back of the wall blocks.

A minimum embedment of 2 feet should be used for the wall blocks, assuming that the adjacent grade is level. If the adjacent grade is sloped, the wall should be additionally embedded until the face of the wall footing is at least 6 feet (horizontally) from the face of the slope.

Wall footings bearing on undisturbed medium dense or denser native soil, Colluvium/Weathered Soil or structural fill as described above can be designed using an average allowable bearing value of 2,500 psf with a maximum toe pressure of 3,500 psf, when the adjacent downhill slope is 4H:1V or flatter. Passive resistance on the embedded portion of the wall should be assumed to be zero.

Final design values for length and spacing of the grids should be specified by the geogrid system manufacturer.

7.11 PAVEMENT SUPPORT

Pavement subgrades for the access and parking areas should be prepared as recommended under sections **7.3 SITE PREPARATION** and **7.4 STRUCTURAL FILL**.

7.12 STORMWATER VAULT

We recommend that the stormwater vault be founded on the native Pre-Olympia Age Glacial Deposits or on structural fill that extends down to the native soils. For foundations supported on surfaces prepared as outlined above, we recommend an average allowable soil bearing pressure of 2,500 psf. Settlement of the vaults when founded as previously described is expected to be reasonably uniform and should be less than ½ inch. Settlement is expected to occur rapidly and should be essentially complete at the end of construction.

The stormwater vault walls will experience lateral soil loads from backfill placed adjacent to the vault walls. The magnitude of lateral loads acting on the vault walls depends on whether or not the walls can yield. If the wall can yield approximately 0.003 times its height, then an active lateral soil pressure is appropriate for design. If yielding is limited to less than 0.003 times the wall height, then at-rest lateral soil pressures should be used for design. Assuming drained conditions behind the wall, the recommended at-rest and active lateral design pressures are 50 and 35 pcf (equivalent fluid pressure), respectively.

When positive drainage cannot be achieved, the walls should be designed for buoyant lateral loading conditions. The resulting lateral pressures including the hydrostatic pressure would be 85 and 80 pcf equivalent fluid for the at-rest and active condition, respectively.

We recommend that the backfill consist of sand or sand and gravel containing less than 5 percent fines. Backfill should be placed in lifts with a maximum loose thickness of about 8 inches and compacted to 90 to 92 percent of the MDD. Only hand-operated compaction equipment should be permitted within 5 feet of the wall to help prevent the development of excessive lateral pressure on the wall caused by the compaction.

If the wall is designed for drained conditions, we recommend that a drain be placed at the base of the wall. The drain should consist of a 4-inch diameter, smooth-walled, perforated PVC pipe that is surrounded by at least 6 inches of clean gravel (clean, washed, rounded gravel with a uniform diameter between ¾ and 1¼ inches) encapsulated in a non-woven geotextile fabric such as Mirafi 140N.

7.13 PERMANENT DRAINAGE MEASURES

7.13.1 General

As previously described, the Apartment Building Area and the Residential Building Area are underlain by relatively pervious Colluvium/Weathered Soil that overlies nearly impermeable Pre-Olympia Age Glacial Deposits. The Pre-Olympia Age Glacial Deposits tend to prevent the downward infiltration of water, and therefore create seasonally perched ground water conditions. Based on the tendency to create perched ground conditions, it is important that appropriate permanent drainage measures be provided for the project. In our opinion, these drainage measures should consist of footing drains and French drains. Also, surface water runoff should be adequately handled.

7.13.2 Footing Drains

Footing drains should be provided for the exterior footings of the buildings. These drains should consist of a 4-inch-diameter, smooth-walled) perforated PVC pipe installed at the outside base of the perimeter footing. The perforated pipe should be embedded in a zone of clean gravel (clean, washed, rounded gravel with a uniform diameter between ¾ and 1¼ inches) and encapsulated in a non-woven geotextile fabric such as Mirafi 140N. The drain pipe should be connected to a tightline system leading to a suitable discharge at appropriate intervals such that water backup does not occur. Cleanouts should be installed at approximate 40 foot intervals.

7.13.3 Interceptor Drains

A system of interceptor drains may be designed and installed in landscaped areas to provide better drainage, especially in lawn and landscape areas. Without this system, these areas may be intermittently saturated during the wet season. The layout of interceptor drains should be a field decision during construction.

The interceptor drain trenches should be at least 24-inches wide and 24-inches deep. Where possible, the trenches should penetrate through the Colluvium/Weathered Soil and extend into the relatively impervious native soils. The base and sides of the interceptor drain trench should be lined with non-woven geotextile fabric such as Mirafi 140N. No fabric should be placed over the top of the trench backfill. It is important that the filter fabric not be placed in a muddy trench to avoid filling the pore spaces with mud and plugging the fabric.

Interceptor drains should be composed of a zone of clean gravel (clean, washed, rounded gravel with a uniform diameter between ¾ and 1¼ inches). Smooth-wall perforated pipe having a minimum diameter of 4 inches should be embedded within this zone of gravel. The drainpipe should be installed with the perforations down and should be sloped to drain. At appropriate intervals, the perforated drainpipe should be connected to a tightline leading to the stormwater collection system for the site. Cleanouts should be installed at appropriate intervals.

7.13.4 Other Drainage Considerations

We recommend that the ground surface be sloped away from building areas to permit drainage away from the foundations. We recommend that roof drains be connected to a tightline leading to the stormwater system. This tightline should be independent from the footing drains.

Appropriate surface swales, drainage ditches and other facilities should be installed to control and collect surface stormwater runoff.

8.0 USE OF THIS REPORT

We have prepared this revised report for use by Vineyards Construction, LLC in evaluating the geotechnical conditions of the subject property. The data and report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

If there are changes in the grades, locations, configurations or types of the facilities planned, the conclusions and recommendations presented in this report may not be applicable. If design changes are made, we request that we be given the opportunity to review our conclusions and recommendations and to provide a written modification or verification. When the design has been finalized, we recommend that the final design and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended.

There are possible variations in subsurface conditions between the explorations and also with time. A contingency for unexpected conditions should be included in the budget and schedule. Sufficient observation, testing and consultation by our firm should be provided during construction to evaluate that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions observed during construction differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

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We trust this revised report meets your present needs. Please call if you have any questions concerning this report.

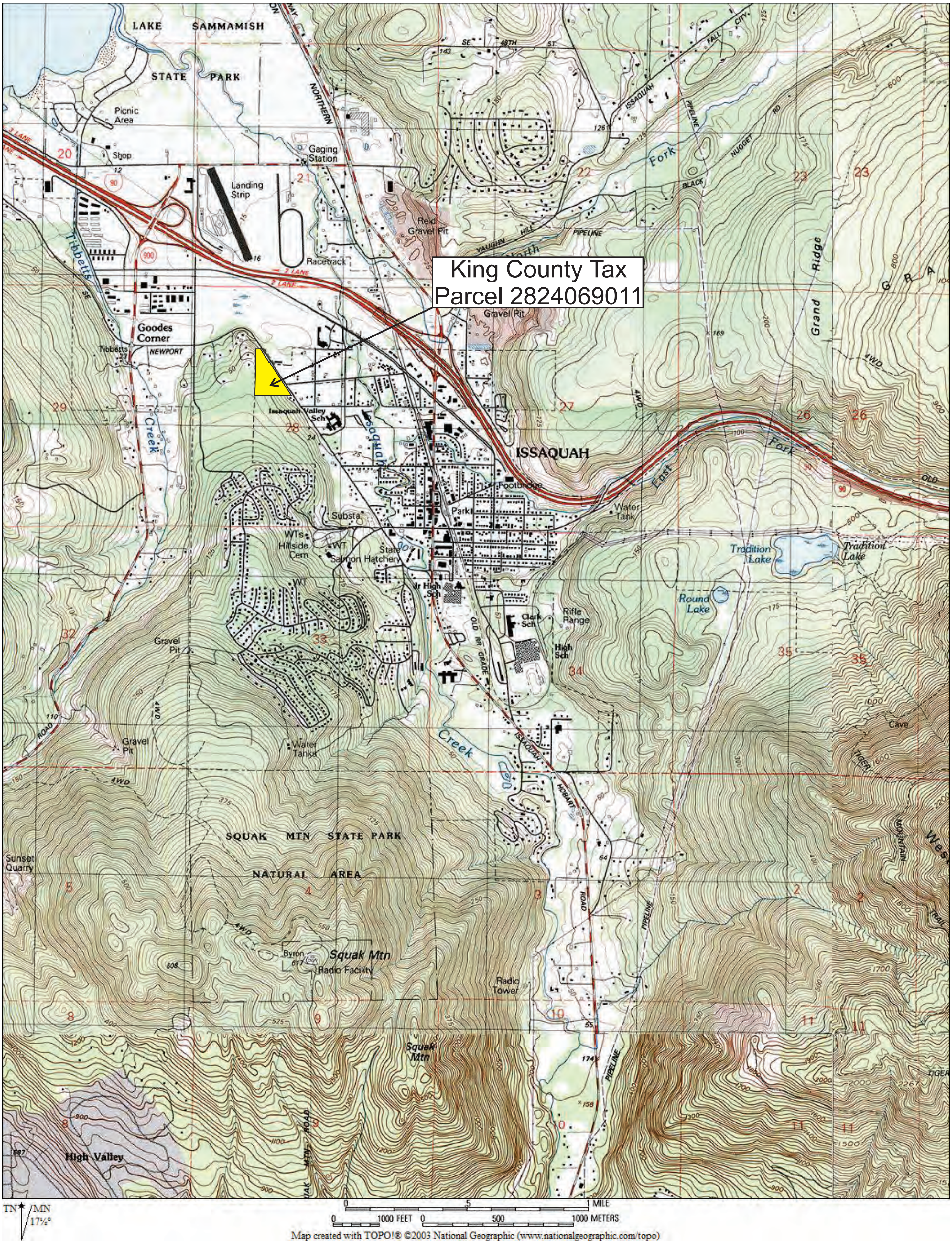
Yours very truly,
Icicle Creek Engineers, Inc.

Kathy S. Killman, LEG
Principal Engineering Geologist

Brian R. Beaman, PE, LEG, LHG
Principal Engineer/Geologist/Hydrogeologist

Document ID: 0520002.revisedreport

FIGURES



ICE File No. 0520-002

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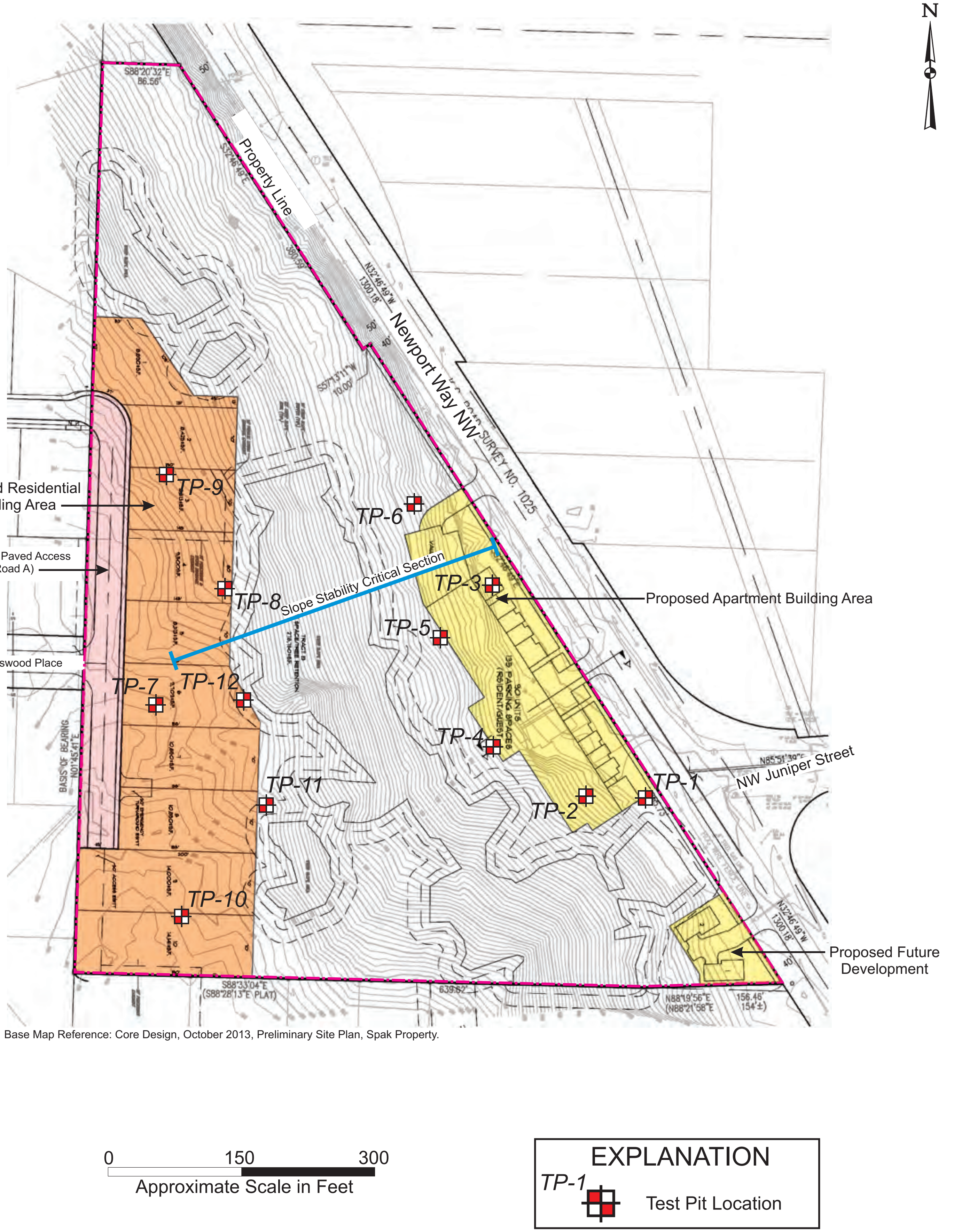
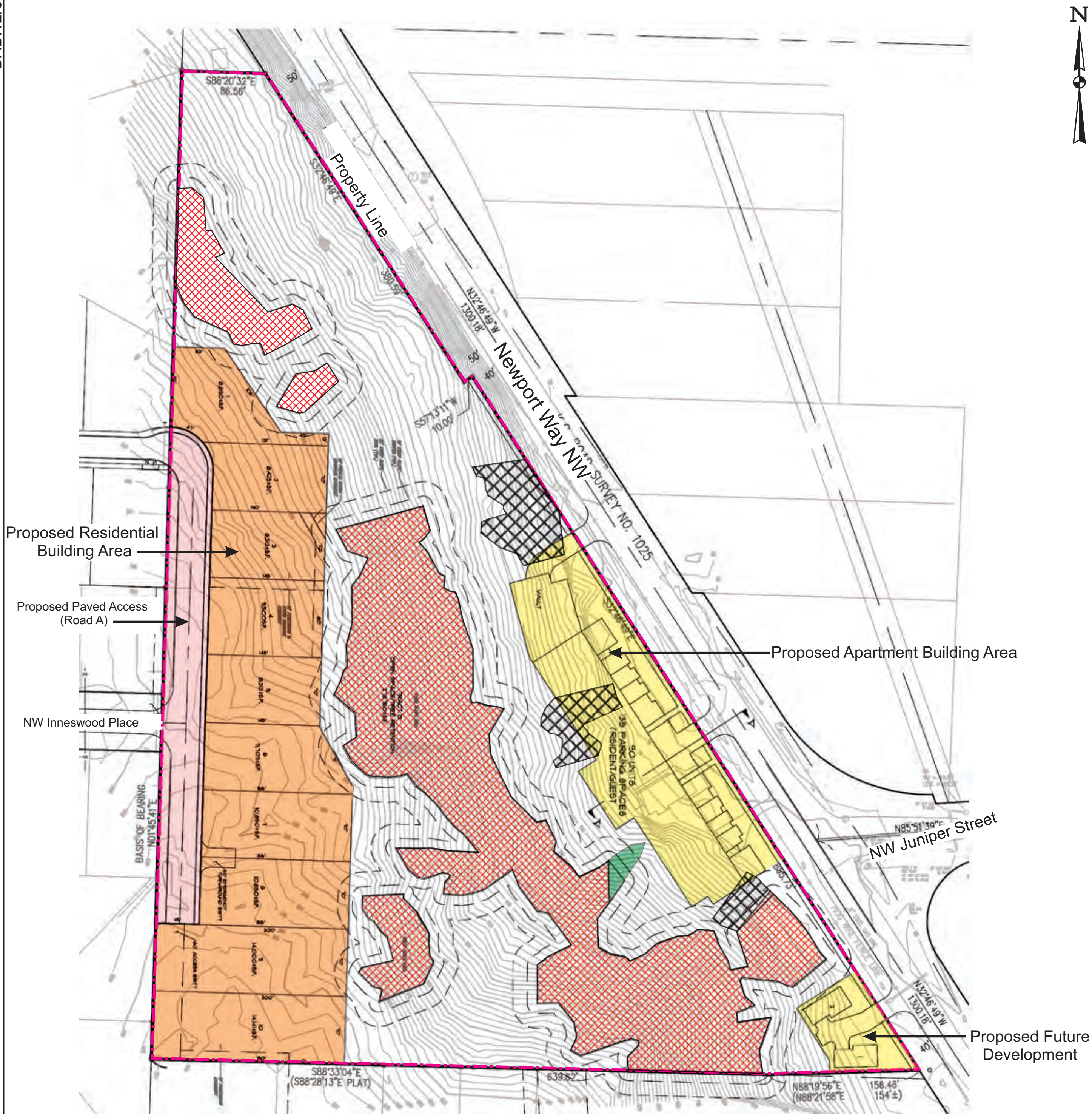
Vicinity Map - Figure 1



Base Map Reference: Core Design, October 2013, Preliminary Site Plan, Spak Property.

Site Plan - Figure 2

BRB:12/31/13



Base Map Reference: Core Design, October 2013, Preliminary Site Plan, Spak Property.

EXPLANATION

Regulated Steep Slope Hazard Areas - slopes inclined more than 40 percent grade and greater than 20 feet in vertical elevation change (LUC Chapter 18.10.390/580.E1).

Exempt Steep Slope Hazard Areas - slopes inclined more than 40 percent grade and created, in part, by previous legal grading activities (LUC Chapter 18.10.390/580.E2).
Exempt steep slope that is less than 20-feet high which is part of a larger steep slope area and topographically in a more favorable setting.

Notes: 1) See report text for additional details.
2) LUC = City of Issaquah Land Use Code.
3) Steep Slope Hazard Area mapping used base map topographic (contour) information.

0 150 300
Approximate Scale in Feet

ICE File No. 0520-002

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Steep Slope Hazards Map - Figure 3

APPENDIX A

SUBSURFACE EXPLORATION PROGRAM

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

Subsurface conditions were explored by excavating twelve test pits (Test Pits TP-1 to TP-12) to depths ranging from about 8 to 15½ feet on March 22 and 23, 2012 using a John Deere 50D mini-excavator owned and operated by T.E. Briggs Construction Company. The approximate locations of the test pits, as shown on the Site Plan, Figure 2, were measured in the field by determining the approximate distance from known site features and by obtaining latitude and longitude coordinates with a hand-held GPS. The test pits were continuously observed by an engineering geologist from ICE who classified the soils, observed ground water conditions, and prepared a detailed log of each test pit. The soil and character described on the test pit logs are based on conditions observed, our experience and judgment, and the difficulty of excavation.

Soils were classified in general accordance with the classification system described in Figure A-1. The test pit logs are presented in Figures A-2 through A-5. The test pit logs are based on our interpretation of the field data and indicate the various types of soils encountered. The logs also indicate the depths at which the soils characteristics change. The densities noted on the test pit logs are based on the difficulty of digging, probing with a ½-inch-diameter steel rod, and our experience and judgment.

The test pits were backfilled upon completion by placing the excavated soil into the test pit in approximate 1-foot thick lifts; each lift was compacted by tamping with the excavator bucket. No other site restoration was completed.

Unified Soil Classification System

MAJOR DIVISIONS			Soil Classification and Generalized Group Description	
Coarse-Grained Soils More than 50% retained on the No. 200 sieve	GRAVEL More than 50% of coarse fraction retained on the No. 4 sieve	CLEAN GRAVEL	GW	Well-graded gravels
			GP	Poorly-graded gravels
		GRAVEL WITH FINES	GM	Gravel and silt mixtures
			GC	Gravel and clay mixtures
	SAND More than 50% of coarse fraction passes the No. 4 sieve	CLEAN SAND	SW	Well-graded sand
			SP	Poorly-graded sand
		SAND WITH FINES	SM	Sand and silt mixtures
			SC	Sand and clay mixtures
Fine-Grained Soils More than 50% passing the No. 200 sieve	SILT AND CLAY Liquid Limit less than 50	INORGANIC	ML	Low-plasticity silts
			CL	Low-plasticity clays
		ORGANIC	OL	Low plasicity organic silts and organic clays
	SILT AND CLAY Liquid Limit greater than 50	INORGANIC	MH	High-plasticity silts
			CH	High-plasticity clays
		ORGANIC	OH	High-plasticity organic silts and organic clays
Highly Organic Soils	Primarily organic matter with organic odor		PT	Peat

Notes: 1) Soil classification based on visual classification of soil in general accordance with ASTM D2488.
2) Soil classification using laboratory tests is based on ASTM D2487.
3) Description of soil density or consistency is based on interpretation of blow count data and/or test data.

Soil Particle Size Definitions

Component	Size Range
Boulders	Greater than 12 inch
Cobbles	3 inch to 12 inch
Gravel	3 inch to No. 4 (4.78 mm)
Coarse	3 inch to 3/4 inch
Fine	3/4 inch to No. 4 (4.78 mm)
Sand	No. 4 (4.78 mm) to No. 200 (0.074mm)
Coarse	No. 4 (4.78 mm) to No. 10 (2.0 mm)
Medium	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Less than No. 200 (0.074 mm)

Soil Moisture Modifiers

Soil Moisture	Description
Dry	Absence of moisture
Moist	Damp, but no visible water
Wet	Visible water

Depth (feet) (1)	Soil Group Symbol (2)	Test Pit Description (3)
Test Pit TP- 1		Approximate Ground Surface Elevation: 73 feet
0.0 - 0.3		Sod and Topsoil
0.3 - 1.0	GM	Dark grayish-brown silty fine GRAVEL with sand (loose, moist) (Fill)
1.0 - 4.0	SM	Mottled light brown and light orangish-brown silty fine to medium SAND with gravel and cobbles (medium dense, moist) (Colluvium/Weathered Soil)
		becomes mottled gray and brown, no cobbles at about 3 feet
4.0 - 6.0	SP-SM	Grayish-brown fine to medium SAND with silt and gravel (dense, moist) (Pre-Olympia Age Glacial Deposits)
6.0 - 9.0	SM	Gray silty fine SAND (dense, moist to wet) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.0 feet on 3/22/12
		Disturbed soil samples obtained at about 4.5 and 6.5 feet
		No caving of the test pit walls observed
		Moderate ground water seepage observed at about 6 feet
Test Pit TP- 2		Approximate Ground Surface Elevation: 85 feet
0.0 - 1.0		Forest Duff and Topsoil
1.0 - 4.5	SM	Brown silty fine SAND with gravel, cobbles, occasional boulders and fine roots (very loose to loose, moist) (Colluvium)
		no roots at about 3 feet
		grades to medium dense at about 4 feet
4.5 - 7.5	SM	Light brownish-gray silty fine to medium SAND with gravel, cobbles and occasional boulders (medium dense, moist) (Weathered Soil)
7.5 - 10.5	SP/ML	Interbedded light gray fine to medium SAND and SILT (dense/very stiff, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 10.5 feet on 3/22/12
		Disturbed soil samples obtained at about 3.0, 5.0 and 9.0 feet
		No caving of the test pit walls observed
		No ground water seepage observed
Test Pit TP- 3		Approximate Ground Surface Elevation: 72 feet
0.0 - 0.3		Sod and Topsoil
0.3 - 1.5	GM	Dark grayish-brown silty fine GRAVEL with sand and abundant fine roots (very loose, moist) (Fill)
1.5 - 3.0	SM	Brown silty fine SAND with gravel and cobbles, and abundant charcoal fragments (loose to medium dense, moist) (Fill)
3.0 - 4.5	SM	Mottled light grayish brown and light orangish-brown silty fine sand with occasional gravel and cobbles (medium dense, moist) (Colluvium)
4.5 - 8.0	GP-GM	Brown fine to coarse GRAVEL with silt, sand and cobbles (dense, moist to wet) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 8.0 feet on 3/22/12
		Disturbed soil samples obtained at about 1.0, 2.0, 4.0 and 6.5 feet
		Slight caving of the test pit walls observed below about 5 feet
		Rapid ground water seepage observed at about 6 feet

See Notes on Figure A-5

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Test Pit Logs - Figure A-2

Depth (feet) (1)	Soil Group Symbol (2)	Test Pit Description (3)
Test Pit TP-4		Approximate Ground Surface Elevation: 91 feet
0.0 - 0.5		Sod and Topsoil
0.5 - 3.5	SM	Orangish-brown silty fine to medium SAND with gravel, cobbles and fine roots (very loose to loose, moist) (Colluvium)
3.5 - 5.5	SP	Light brownish-gray fine to medium SAND with a trace of silt (loose to medium dense) (Weathered Soil)
5.5 - 12.0	SP	Light gray fine to medium SAND with occasional gravel and cobbles (dense, moist) (Pre-Olympia Age Glacial Deposits) interbeds of light-gray hard SILT observed at about 11 to 12 feet
12.0 - 15.5	SM	Light gray silty fine to medium SAND with gravel and cobbles (very dense, moist to wet) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 15.5 feet on 3/22/12 Disturbed soil samples obtained at 2.0, 4.0 , 6.5 and 9.0 feet Moderate caving of the test pit walls from about 0 to 4 feet Slow ground water seepage observed at about 13 and 15 feet
Test Pit TP- 5		Approximate Ground Surface Elevation: 98 feet
0.0 - 0.8		Forest Duff and Topsoil
0.8 - 5.0	GM	Orangish-brown silty fine to coarse GRAVEL with sand, cobbles and abundant fine roots (loose to medium dense, moist) (Colluvium/Weathered Soil) no roots at about 4 feet
5.0 - 8.5	GM	Light brownish-gray silty fine to coarse GRAVEL with sand (very dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 8.5 feet on 3/22/12 No caving of the test pit walls observed No ground water seepage observed
Test Pit TP-6		Approximate Ground Surface Elevation: 95 feet
0.0 - 0.8		Forest Duff and Topsoil
0.8 -3.0	SM	Orangish-brown silty fine SAND with gravel, cobbles and abundant roots (loose to medium dense, moist) (Colluvium/Weathered Soil)
3.0 - 8.5	SM	Light brownish-gray silty fine SAND with gravel, cobbles and occasional boulders (very dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 8.5 feet on 3/22/12 No caving of the test pit walls observed No ground water seepage observed

See Notes on Figure A-5
0520002/123113

Depth (feet) (1)	Soil Group Symbol (2)	Test Pit Description (3)
Test Pit TP-7		Approximate Ground Surface Elevation: 201 feet
0.0 - 0.5		Forest Duff and Topsoil
0.5 - 2.0	ML	Light brown sandy SILT and SILT with abundant roots (soft, moist) (Weathered Soil)
2.0 - 6.0	ML/MH	Mottled light grayish-brown and light orangish-brown SILT with variable amounts of clay (medium stiff to stiff, moist) (Weathered Soil)
6.0 - 10.0	ML	Mottled light grayish-brown and light orangish-brown SILT (very stiff, moist) (Pre-Olympia Age Glacial Deposits) grades to hard sandy SILT at about 9 feet
		Test pit completed at approximately 10.0 feet on 3/23/12
		Disturbed soil samples obtained at 5.0 and 9.0 feet
		No caving of the test pit walls observed
		Slow ground water seepage observed at about 4 feet
Test Pit TP- 8		Approximate Ground Surface Elevation: 180 feet
0.0 - 0.8		Forest Duff and Topsoil
0.8 - 3.0	SM	Brown silty fine SAND with occasional gravel and abundant roots (loose, moist to wet) (Weathered Soil)
3.0 - 5.0	ML	Mottled light grayish-brown and light orangish-brown SILT with fine sand (stiff, moist) (Weathered Soil)
5.0 - 6.0	ML	Light brownish-grayish brown sandy SILT (very stiff, moist) (Pre-Olympia Age Glacial Deposits)
6.0 - 9.0	SP	Light grayish-brown medium SAND with a trace of silt and gravel (dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.0 feet on 3/23/12
		Disturbed soil sample obtained at 7.0 feet
		No caving of the test pit walls observed
		Moderate ground water seepage observed at about 3 feet
Test Pit TP-9		Approximate Ground Surface Elevation: 179 feet
0.0 - 0.5		Forest Duff and Topsoil
0.5 - 3.5	SM	Brown silty fine SAND with a trace of gravel and roots (very loose to loose, moist to wet) (Weathered Soil)
3.5 - 6.0	ML	Mottled light grayish-brown and light orangish brown SILT with fine sand (medium stiff to stiff, moist) (Weathered Soil)
6.0 - 7.0	ML	Light grayish-brown sandy SILT (stiff, moist) (Pre-Olympia Age Glacial Deposits)
7.0 - 9.0	SM	Light grayish-brown silty fine SAND with a trace of gravel (dense to very dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.0 feet on 3/23/12
		No caving of the test pit walls observed
		Slow ground water seepage observed at about 3.5 feet

See Notes on Figure A-5

0520002/123113

Depth (feet) (1)	Soil Group Symbol (2)	Test Pit Description (3)
Test Pit TP-10		Approximate Ground Surface Elevation: 213 feet
0.0 - 0.5		Forest Duff and Topsoil
0.5 - 1.5	SM	Light brown silty fine SAND with abundant roots (loose, moist) (Weathered Soil)
1.5 -3.0	ML/SM	Mottled light gray and light orangish-brown SILT and silty fine SAND (stiff/medium dense, moist) (Weathered Soil)
3.0 - 9.0	ML/SM	Mottled light grayish-brown and light orangish-brown SILT and silty fine SAND (very stiff/dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.0 feet on 3/23/12
		No caving of the test pit walls observed
		No ground water seepage observed
Test Pit TP- 11		Approximate Ground Surface Elevation: 193 feet
0.0 - 0.5		Forest Duff and Topsoil
0.5 - 2.5	SM	Orangish-brown silty fine SAND with occasional gravel and abundant roots (loose, moist) (Weathered Soil)
2.5 - 4.0	SM	Mottled light grayish-brown and light orangish-brown silty fine SAND with occasional gravel (medium dense to dense, moist) (Weathered Soil)
4.0 - 9.0	SM	Light brownish-gray silty fine SAND with occasional gravel (very dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.0 feet on 3/23/12
		No caving of the test pit walls observed
		No ground water seepage observed
Test Pit TP-12		Approximate Ground Surface Elevation: 189 feet
0.0 - 1.0		Forest Duff and Topsoil
1.0 - 2.5	SM	Light brown silty fine SAND with abundant roots (loose, moist) (Weathered Soil)
2.5 - 3.5	SM	Mottled light grayish-brown and light orangish-brown silty fine SAND with roots (medium dense, moist) (Weathered Soil)
3.5 - 7.0	SP-SM	Light brown fine SAND with silt (dense, moist) (Pre-Olympia Age Glacial Deposits)
7.0 - 9.5	SM	Light brownish-gray silty fine SAND with occasional gravel (dense to very dense, moist) (Pre-Olympia Age Glacial Deposits)
		Test pit completed at approximately 9.5 feet on 3/23/12
		No caving of the test pit walls observed
		No ground water seepage observed

Notes:

(1) The depths on the test pit logs are shown in 0.5 foot increments, however these depths are based on approximate measurements across the length of the test pit and should be considered accurate to 1.0 foot. The depths are relative to the adjacent ground surface.

(2) The soil group symbols are based on the Soil Classification System, Figure 3.

(3) The approximate test pit locations are shown on the Site Plan, Figure 2.

0520002/123113

ICICLE CREEK
ENGINEERS
Geotechnical, Geological and Environmental Services

Technical Memorandum

To: Robert P. Wenzl, Managing Member, Inneswood Estates LLC

From: Brian R. Beaman, PE, LEG, LHG

State of Washington Licensed Geologist Engineering Geologist 671 BRIAN R. BEAMAN

BRIAN R. BEAMAN STATE OF WASHINGTON 24010 REGISTERED PROFESSIONAL ENGINEER EXPIRES 10-30-14

Date: June 2, 2014

ICE File No: 0520-002

Subject: Deep-Seated Landslide Evaluation - Slope Stability Analysis
Inneswood Estates, King County Parcel 2824069011
Issaquah, Washington

At the request of Cliff Williams of Development Management Engineers, LLC, Icicle Creek Engineers (ICE) is providing additional slope stability analysis for Inneswood Estates, a proposed residential development located at 905 Newport Way NW in Issaquah, Washington (King County Parcel No. 2824069011). ICE previously completed a geotechnical evaluation of the approximately 10.5 acre property (formerly referred to as the Spak property) which included detailed analysis of shallow slope failure (the expected mechanism of failure) in a steep slope area; the results are presented in our report dated December 31, 2013.

Our services are being provided in response to comments from the City of Issaquah (City) as a result of third-party review of ICE's December 31, 2013 report. These comments were discussed by phone with Doug Schlepp, PE, of RH2 Engineering, the City's reviewer, and with Chris Breeds, PE, geotechnical engineer with SubTerra, Inc., the City's third-party reviewer. Mr. Williams requested that ICE respond to the City's comments by completing deep-seated failure analysis at the identified critical section in the hillside area.

As described in ICE's December 31, 2013 report, a shallow slope failure such as a debris slide is the most likely mechanism of mass wasting processes at the site; this shallow slope failure was quantitatively analyzed in ICE's December 31, 2013 report (page 6, Section 6.2.5). A deep-seated landslide is low risk is the site area, considering the local geologic conditions. However, the City's third-party reviewer has requested that this analysis be completed.

The slope stability analysis was completed using the computer application Slide 6.0 (RocScience, April 25, 2014, Version 6.029). This computer application has the capability of comprehensive slope stability analysis along with sensitivity and probabilistic analysis. Slope stability analysis was completed using the Bishop Simplified Method under static and dynamic (seismic/earthquake) conditions. The analysis for deep-seated landslide failure was completed at the "Slope Stability Critical Section" shown on Figure 2 of ICE's December 31, 2013 report (a copy of Figure 2 is attached to this Technical Memorandum).

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Soil strength parameters were obtained from standard values that have been assigned to the geologic conditions expected. The following is a summary of the soil strength parameters used in our analysis.

Soil Type	Moist Unit Weight (pcf)	Φ (degrees)	C (psf)
Weathered Soil	115	30	0
Pre-Olympia Age Glacial Deposits	120	36	200

Φ = angle of internal friction
C = cohesion
pcf = pounds per cubic foot
psf = pounds per square foot

For earthquake loading we used one-half of the horizontal peak ground acceleration of 0.35g for a 10 percent in 50 years return period. The horizontal peak ground acceleration was obtained from the US Geological Survey (http://earthquake.usgs.gov/hazards/).

In stability analyses, the relative stability of a slope is expressed in terms of a factor of safety (FOS) against sliding for the most likely potential failure surface. A FOS of 1.0 corresponds to the conditions in which the resisting and the driving forces are equal (equilibrium conditions), and failure would theoretically be imminent as the result of a decrease in the resisting force or an increase in the driving force. A FOS greater than 1.0 indicates that the forces tending to resist sliding are greater than the forces tending to cause sliding.

For the purpose of this analysis, we evaluated two conditions for 1) static and 2) dynamic (seismic) stability. The generalized soil conditions included about 5 feet of Weathered Soil overlying Pre-Olympia Age Glacial Deposits as described in ICE's December 31, 2013 report. Ground water emerges near or at the toe of the slope (this has generally been observed in the field behind the existing house), with a relatively steep gradient up into the slope as a conservative assumption.

The slope stability analyses (FOS) for static and seismic conditions are summarized in the following table.

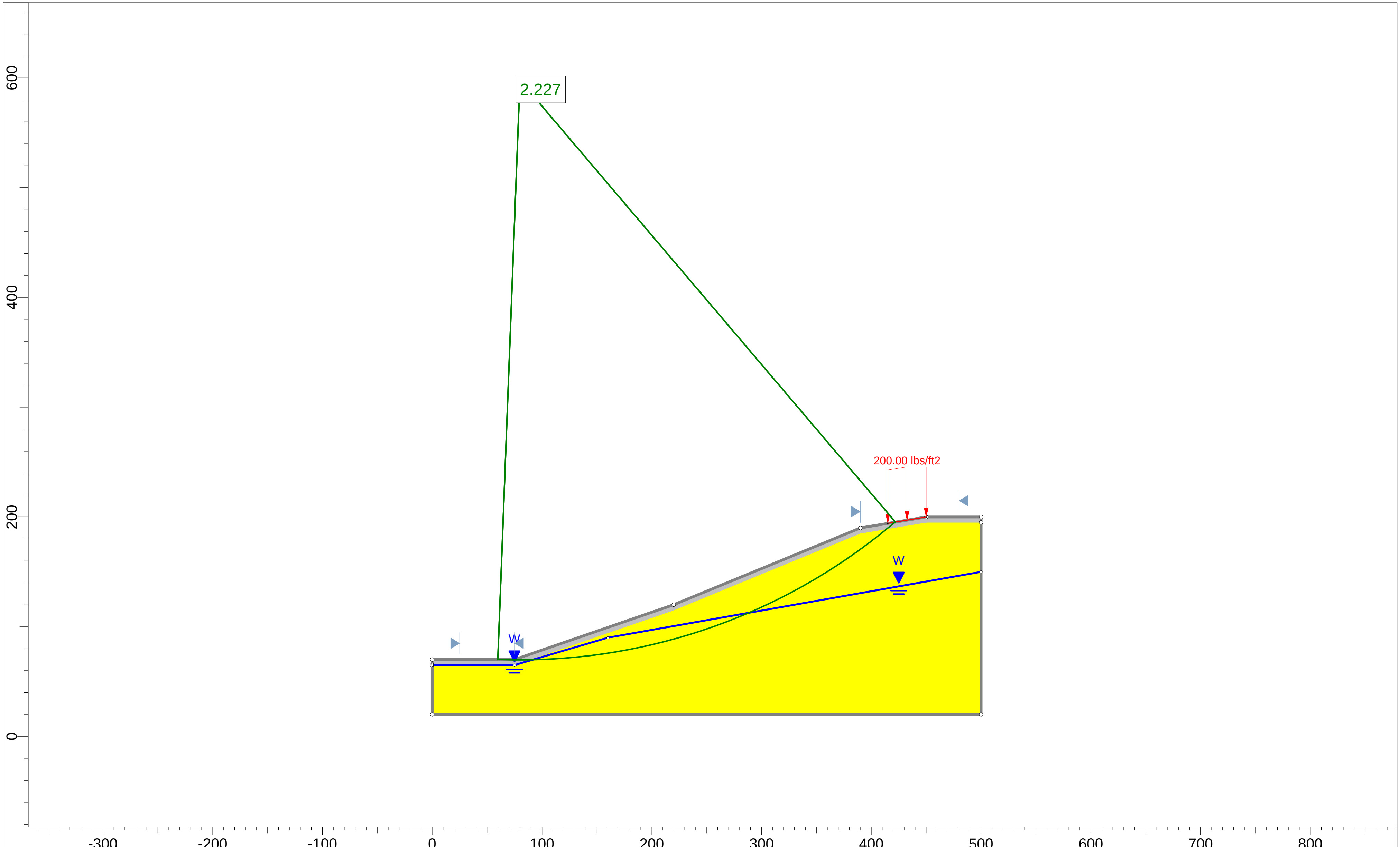
Static Condition FOS	Seismic Condition FOS
2.23	1.44

In our opinion, the FOS is acceptable for the site conditions and expected use (FOS greater than 1.5 under static condition and greater than 1.1 under seismic condition)

Attachments: Site Plan – Figure 2 (from ICE December 31, 2013 report)
Inneswood Property – KC Parcel No. 2824069011 - Slope Stability Analysis (Static Condition) – Attachment 1
Inneswood Property – KC Parcel No. 2824069011 - Slope Stability Analysis (Seismic Condition) – Attachment 2

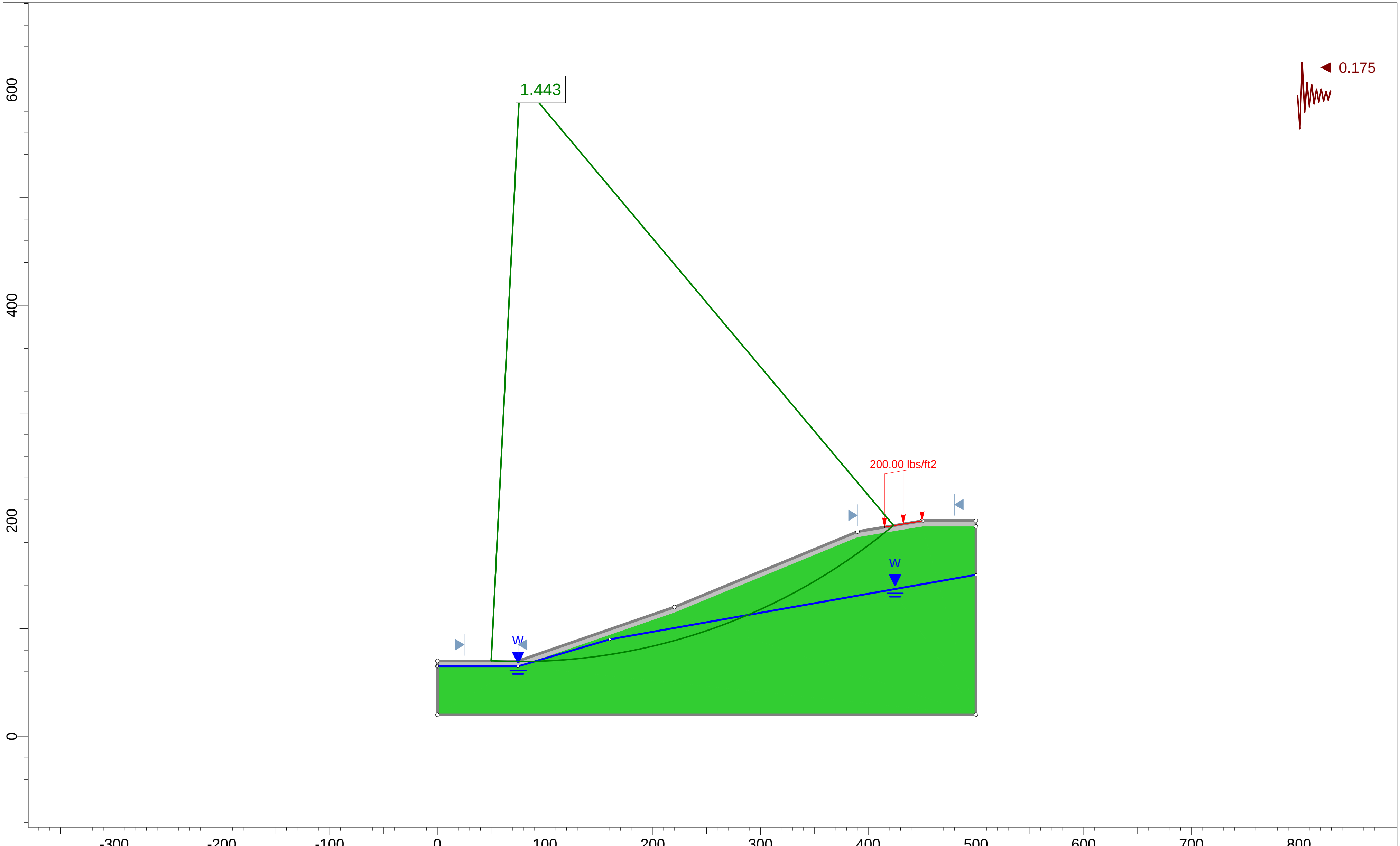
Submitted via email (PDF) and surface mail (one original copy)
cc: Cliff Williams, Development Management Engineers, LLC (email and three original copies)

Base Map Reference: Core Design, October 2013, Preliminary Site Plan, Spak Property.



Project	Inneswood Estates - KC Parcel No. 2824069011 - Slope Stability Analysis (Static Condition) - Attachment 1		
Analysis Description	DEEP-SEATED - STATIC		
Drawn By Brian Beaman	*Scale* 1:1451	*Company*	Icicle Creek Engineers
Date	5/21/2014	*File Name*	0520002.Static.slim

SLIDEINTERPRET 6.029



Project	Inneswood Estates - KC Parcel No. 2824069011 - Slope Stability Analysis (Seismic Condition) - Attachment 2
Analysis Description	DEEP-SEATED - SEISMIC
Drawn By	Brian Beaman
Scale	1:1479
Company	Icicle Creek Engineers
Date	5/21/2014
File Name	0520002.Seismic.slim

rocscience

SLIDEINTERPRET 6.029

ICICLECREEK ENGINEERS

Technical Memorandum

To: Robert Wenzl, Managing Member
Inneswood Estates, LLC

From: Brian R. Beaman, PE, LEG, LHG

Date: May 3, 2016

ICE File No: 0520-002

Subject: Critical Areas Study, Coal Mine Hazard Evaluation
Inneswood Apartments, Inneswood Estates, LLC
King County Parcel Nos. 282406-9011 and 282406-9395
Issaquah, Washington

At your request, Icicle Creek Engineers (ICE) completed an evaluation of coal mine hazards at the Inneswood Apartments project for Inneswood Estates, LLC located at King County Parcel Nos. 282406-9011 and 282406-9395 in Issaquah, Washington. We understand that these parcels are proposed for multi-family (apartments) use. ICE previously completed geotechnical evaluations of the property related to overall project development considerations, slope stability of steep slope hazard areas, storm drain hillside installation and a stormwater vault; the results are presented in our reports and technical memorandums dated December 31, 2013, June 2, 2014, June 2, 2014 and July 22, 2015, respectively.

We understand that the City of Issaquah has requested that coal mine hazards be evaluated for this project (Issaquah Municipal Code Chapters 18.10.390 and 18.10.520 A.).

ICE has considerable experience in the evaluation of coal mine hazards in the Issaquah area. Based on review of our in-house library, the Washington State Department of Natural Resources Coal Mine Map database (http://www.dnr.wa.gov/geologyportal) and our experience in this part of Issaquah, the nearest abandoned underground coal mines are located approximately 3,600 feet (0.7 miles) to 4,250 feet (0.8 miles) southeast of the Inneswood Apartments project area.

In our opinion, no coal mine hazards exist within the Inneswood Apartments project area.

Submitted via email (pdf) and surface mail (two original copies)
cc: Cliff Williams, PE, Development Management Engineers, LLC (email)